AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

No. 901.

STEEL-CONCRETE CONSTRUCTION.

An Informal Discussion at the Annual Convention, June 26th, 1901.

SUBJECT FOR DISCUSSION:

"Steel-Concrete Construction.
"What stress in tension and compression should be allowed in concrete?
"What is the proper modulus of elasticity of concrete?
"In Steel-Concrete Arches:
"(1) What should be the ratio of steel section to concrete section, and what is the best form and disposition of the former?
"(2) What consideration should be given to temperature changes and consequent stresses?
"(3) What are the best proportions for concrete, and what is the best method of placing it?"

By Messrs. R. S. BUCK, L. S. MOISSEIFF, J. R. WORCESTER, H. S. JACOBY, LEONARD C. WASON, A. L. JOHNSON, WILLIAM PARKER and WILLIAM B. FULLER.

Mr. Buck.

R. S. BUCK, M. Am. Soc. C. E.—There are still doubts in some quarters, not only reputable but excellent professionally, as to the thorough soundness of steel-concrete construction for arch bridges.

The extent to which there is just cause for such doubts, and whether or not the cause can be removed by revision of practice, it is hoped this discussion will assist in disclosing. The speaker himself does not hope to contribute more than the suggestion of a few points,

Mr. Buck. which, in his limited experience with the system, struck him as being in need of discussion and elucidation.

Steel-concrete construction in arches has, both in Europe and this country, passed from the experimental stage to that of general practical use.

Although there is still lacking the crucial test of time, the large number of these bridges now in service seems satisfactory, and there appear no reports of failure attributable to any inherent weakness of the system, despite some remarkably bold dealings with it.

If, as now appears reasonably certain, the system is not only here to stay, but is having its field of application steadily enlarged, it is surprising that there are not more working data on the subject, of certain scientific origin, in place of the variable "rule-of-thumb" methods which seem now to govern in American practice.

European engineers have given the subject much more careful study, and have conducted some very elaborate experiments to settle certain points. But the results of these experiments have reached us here only in very general terms, or else are buried in unfamiliar foreign publications, from which they have not yet been mined. That there is enough in the records of European practice, investigation and experiment, to frame a complete, reliable scientific code of procedure for this system, is doubtful; but there can be no doubt that they would contribute largely to this end.

Mr. Fr. von Emperger, on April 4th, 1894, read a paper entitled "The Development and Recent Improvement of Concrete-Iron Highway Bridges."* This was, perhaps, the first general introduction of the subject in this country, and with the exception of an article† by Edwin Thacher, M. Am. Soc. C. E., on the Paterson Bridge, this remains, so far as the speaker knows, the only attempt at analytical treatment of the system indulged in in this country. Mr. von Emperger's paper lacks many things, to render it of scientific or practical value, and was evidently intended rather to awaken interest in the possibilities of the system than to assist in establishing any laws whereby these possibilities could be confidently attained. It made known, generally, the fact that the Austrian Society of Engineers and Architects had conducted an elaborate and doubtless valuable series of experiments on metal-concrete arches, and stated that a report would be made on the same, probably at an early date. But this report has not, to the speaker's knowledge, either as a whole or in part, been published in English. It would be a most valuable contribution to our fund of knowledge if the report were so published, and the same supplemented by discussion and experiment along the lines of American practice.

* *Transactions*, Am. Soc. C. E., Vol. xxxi, p. 438.

† *Engineering News*, Sept. 1st, 1899.

Mr. Buck.

As previously stated, there is a great deal of "rule-of-thumb" about American practice in this system, and its general acceptance cannot well come until such methods are replaced by others as carefully grounded as those governing the use of steel.

The engineers of this country have been slow in taking up this system, probably unduly so when its value is fairly considered. Its development has been largely in the hands of contractors and contracting engineers, with whom a vital point has been to bring it in competition with all-steel construction. In this, the speaker inclines to think, there has been rather an unfortunate disposition to reduce the cost unduly at the sacrifice of certainty, stability and durability. It is not possible to construct a steel-concrete arch that will compare in cost with an all-steel bridge and at the same time furnish the desired characteristics to be reasonably looked for in a masonry arch.

The steel-concrete bridge possesses sufficient merits, the speaker believes, when designed on conservative lines, to often reduce the question of first cost to a minor position; and when it cannot do this it is a good form of bridge to let alone.

The combination of concrete and steel unquestionably is one of great mutual benefit, especially to the concrete, but it is very questionable, to the speaker's mind, if this general proposition warrants the high unit stresses allowed on the strength of certain combinations.

On steel 20 000 lbs. per square inch; 500 lbs. per square inch compression on concrete, without allowance for temperature, which may become 1 000 lbs. per square inch under the same rules for fixing sections where a reasonable allowance for temperature is made; 50 to 100 lbs. per square inch in tension in concrete without consideration of temperature, and which, considering temperature, may become 200 lbs. per square inch; are allowances that may appear to meet demands, but, the speaker believes, are hardly justifiable.

The freak bridges built for experimental purposes or for pedestrian traffic in Europe, some of which Mr. von Emperger mentions in his paper, and the fact that these have stood the test of a few years, seem to warrant too much imposition on the system.

An arch of 132 ft. span, $14\frac{3}{4}$ ft. rise and $9\frac{7}{8}$ in. crown thickness (one of those mentioned by Mr. von Emperger, and claimed by him to be designed for a load of 200 lbs. per square foot) would seem to justify much, but until the speaker sees the 200 lbs. per square foot safely applied, and the test of years of service, he will not count such construction, or even a remote approach to it, good. Yet it is not improbable that in European practice such cases as this have to some extent influenced American practice, and in a measure justified thinning down sections when other justification was lacking. The differ-

Mr. Buck. ence between experiment and practice, and different kinds of service, demand very careful consideration. Concrete, despite its versatility and endurance, cannot be relied upon with the same confidence as steel, any more than can any other form of masonry, especially where tension is to be dealt with.

It is rather a striking exemplification of the uncertainty of our knowledge of the action of concrete in conjunction with steel, in arches or elsewhere, that there is such a large variation of opinion as to the correct modulus of elasticity of concrete. This ranges from 700 000 to over 3 000 000. This difference is of course largely due to the variation in composition of the concrete experimented with, but there should not be such a difference in the concrete suitable for arch rings—say Portland cement concrete in proportions of 1:2½:5. The modulus of elasticity of such concrete, within narrow limits, can and should be established much more definitely.

Directly contingent upon this modulus is the correct proportion of concrete to steel capable of taking calculable stresses in the arch ring, and in fact the entire theory of arch-stress calculation.

However, the correct combination of concrete and steel is probably quite as much dependent upon the thorough bonding of the two in order to secure unity of action as upon the capacity of the steel to pick up and carry to the abutments a specific amount of stress.

The correct function of the steel is rather to insure a thorough distribution of the strain in the concrete, and to prevent undue concentration of strain and consequent rupture at some point where there is an element of weakness akin to a nick or other flaw in a steel bar.

The steel should be so disposed as to bond the concrete in three directions, parallel or approximately so to the axis of the ring, normal to the soffit, and transversely normal to the axial plane of the ring; and so proportioned in each direction as to meet all possible demands upon it. Further, the units into which the steel is divided should be such as to insure its reaching all parts of the concrete with its influence.

In this respect the Monier system, while it apparently lacks the steel bonding normal to the soffit of the ring, possesses the advantage of fine division and thorough dissemination and bonding in the other two directions. The Melan system, with steel I-sections, bonds essentially normal and parallel to the intrados, but not transversely. The units of steel in this system are large, and the dissemination through the concrete not so thorough as in the Monier system. In fact, it may be fairly questioned whether this is sufficient to bring all the concrete properly within the range of influence of the steel.

The Thacher system, as originally adopted, comprising steel in the shape of flat bars parallel to the intrados and extrados of the ring, bonded the concrete only in one direction; it being assumed that the steel could do best service as flanges on these lines, and that no

other bonding was necessary. This system is susceptible of ready modification, which will secure bonding in all these directions, and the reduction in size and multiplication of number of units is practically limited to the space necessary for laying the concrete. Mr. Buck.

The treatment of temperature stresses, or at least their non-treatment, is rather an anomalous condition found in dealing with steel-concrete arches.

Mathematically calculated on clearly established proper lines, even with a limited range of about 30° Fahr., the resulting temperature stresses are decidedly startling, especially in flat arches. Although the correctness of the mathematical results is not questioned, it is often deemed expedient and safe to ignore them. The justification for this seems to be that the presence of the steel will prevent serious results if a crack should occur, and that many arches subjected to heavy temperature stresses stand without manifesting weakness. This reasoning is convenient, but hardly scientific or logical. This feature of the system, perhaps more than any other, is in need of thorough discussion.

The question of ingredients and the degree of wetness of the concrete in the arch ring is a very general one, but of much importance, and well worth discussion here.

Another point of importance is the method of laying in continuous rings from abutment to abutment or in large voussoirs. There is now a clear cut difference of opinion on this point, which is also well worth discussion.

Aside from the question of the correct theory governing the sections and proportions of a steel-concrete arch, the speaker inclines to the opinion that there is much room for improvement in the practical details of construction in American practice. Strength of centering and precautions against sinking; uniformity and sufficiency in ramming; careful bonding of fresh and set concrete, and care in securing monolithic construction; care in securing actual contact and adhesion between the concrete and the steel, are all points wherein practice can be materially improved. Ramming is perhaps one of the weakest points in steel-concrete construction, but the speaker looks with reviving hope to the pneumatic rammer which has recently been put upon the market. Some information as to its performance would be most welcome, and it is hard to believe that it will not be an immense improvement on the machine now in use, the cardinal principle of which seems to be to make as little impression on the concrete as possible without loss of its job.

L. S. Moisseiff, Assoc. M. Am. Soc. C. E. (by letter).—*Allowable Tensile and Compressive Stresses.*—With the usual proportions of Portland cement concrete of 1 : 2 : 4 and 1 : 3 : 5, and careful manipulation in making the concrete, a maximum compressive stress of 500 lbs. per Mr. Moisseiff.

Mr. Moisseiff. square inch and a maximum tensile stress of 60 lbs. per square inch may be allowed in beams and arches. A study of a great number of tests made in the United States, as well as in Europe, will show that these values are well within the limits of good engineering. It should be understood, however, that the greatest allowable unit stresses are really meant to be the maximum stresses which will occur in any section of the structure under any vertical or horizontal load, or other agency that can be reasonably expected to act on the same. For arches, stresses due to variations of temperature or to displacements of abutments should be treated as secondary stresses, their amounts should be determined, and they should be provided for within the greatest allowable unit stresses. Then only will the fixing of some limit of unit stress have a definite meaning. The secondary stresses will vary greatly with the proportions of the arches Generally, the flatter the arch, the greater will be the effects of temperature and settling; and for the flat arches the secondaries will show unexpectedly high values. Only by adding the secondary to the so-called primary stresses due to external loads can the maximum stresses acting in any part of the structure be determined. For the stresses so found, the structure should be proportioned by using the fixed maximum unit stress.

The purpose of the structure should also be taken into consideration. A light structure may be used for a highway bridge, with its light loads, but a railroad bridge, with its heavy impact, requires mass, in addition to strength, to absorb the vibrations due to rapidly moving trains. This may sometimes lead to greater thickness of arch than would be absolutely made necessary by the allowable unit stresses. Such increased thickness represents material well placed.

Modulus of Elasticity of Concrete.—The modulus of elasticity of concrete not reinforced by steel must be known in order to determine the secondary stresses. The basis of all formulas or graphical methods used to determine these stresses must be the elastic theory of arches. The famous tests made by the Austrian Society of Engineers and Architects have proved definitely that the elastic theory holds true for concrete arches as well as for any other kind of arches. This necessarily introduces the elastic behavior of the material, and with it the modulus of elasticity, into the determination of all stresses. But, whether the elastic theory or the static assumptions are used for finding the main stresses, the modulus of elasticity must be known, in order to determine the stresses due to displacements and to changes of temperature and volume.

For steel-concrete construction, the value of the modulus of the concrete is of prime importance. To these constructions, steel in the form of wire, bars or shapes is imbedded in a concrete matrix, forming, by the adhesion of the two constituents, a practically new material.

Mr. Moisseiff.

To have a clear understanding of the stresses in such a combination, the elastic behavior of the constituent parts under any stress must be thoroughly known. The modulus of elasticity represents the reciprocal of the elongation per unit stress and unit length, and its value indicates the story of the behavior of the material. In fact, the theory of composite structures, as it has been applied to steel-concrete, is directly dependent on the values of the moduli of elasticity of the two materials.

It has been shown by a wide range of experiments, especially by those made by Professor C. Bach, at the Royal Testing Laboratory at Stuttgart, that Hooke's law of the proportionality between stress and deformation does not hold true. It is very nearly true for steel and wrought iron, but for concrete the curve of elongations has a pronounced curvature, showing that the elongation increases in a higher degree than the stress. The modulus of elasticity of Portland cement concrete is then a variable, decreasing with the increase of unit stress. This characteristic of the elastic behavior of concrete is of importance in steel-concrete construction. It is evident that the more the concrete elongates per unit stress, the modulus of elasticity of the steel being practically constant, the more stress will be carried by the steel.

When the theory of composite structures was first applied to steel-concrete by Professor J. Melan he assumed the modulus of elasticity of Portland cement concrete to be 710 000 lbs. per square inch. This is about one-fortieth of the modulus of steel. This means that the stress in the steel will be forty times the stress in the surrounding concrete. But, even then, the steel will only be stressed in tension to $60 \times 40 =$ 2 400 lbs. per square inch. This is certainly a ridiculously small value for the tensile stress in steel. Since that time it has been found that the value $E = 700\,000$ lbs. for concrete is far below its actual value, and in recent years prominent engineers in this country have used $E =$ 1 400 000 lbs. This gives a ratio of 1 : 20. According to this assumption, the work of the steel is reduced by half.

A study of a great number of tests shows that the modulus of elasticity for Portland cement concrete made in the proportions between 1 : 2 : 4 and 1 : 3 : 5 is from 2 800 000 to 3 500 000 lbs. This makes the modulus of concrete one-tenth to one-eighth that of steel. This new value of the modulus for steel-concrete further reduces the proportion of the work done by the steel.

The question then arises, do we not pay too much for the little the steel does? This seems to be so, according to the accepted theory, but, fortunately, practice is here more correct than theory; in fact, too much should not have been expected of a theory based on pure assumptions. The observations made on numerous steel-concrete beams and arches show that the concrete is stressed considerably beyond any reasonable stress, or otherwise the steel is doing much

Mr. Moisseiff. more than is attributed to it by the composite-structure theory. Only a great number of careful experiments on steel-concrete combinations, in conditions as near as possible to those met in actual practice, can furnish reliable and sufficient data on which to base an adequate theory.

A series of careful experiments on the influence of imbedded metal on the properties of mortar and concrete was made in 1898 by the eminent French engineer, M. Considère, at the laboratory of the Ecole des Ponts et Chaussées. These tests have shown remarkable results. They have proved that concrete in combination with steel is able to elongate considerably more, without impairing its resisting value, than if not reinforced by steel. The same phenomenon has been observed in the experiments made at the same time for the X bridge at Mans, France, by the Chief Engineer, M. Harel de la Noë. This is also confirmed by observations made on arches and by computations made on observed deflections. It is out of place here to give this interesting subject more space.

The writer is at work on a paper, to be presented to the Society at an early date, on "The Theory of Steel-Concrete Structures," and this discussion is merely a brief abstract of its main features. Whatever the explanation of the different behavior of concrete when reinforced by steel from that when not reinforced, it is certain that the theory of concrete structures needs revision. Formulas based on the actual behavior of concrete and steel in combination should be deduced. These formulas will have to account rationally for the relative work done by the two materials. They will have to furnish means to evaluate the capacity of the steel and give the right proportion of steel to concrete.

Ratio of Steel Section to Concrete Section.—From the foregoing discussion it will be seen that no *a priori* ratio of steel section to concrete can be fixed. From some tentative computations, the results seem to require the section of the steel to be between 1 to 1.5% of that of the concrete. The steel should be as well distributed in the concrete as possible, reaching the greater part of the outside layers, and thus increasing the strengthening effect. It should not be nearer to the surface than 3 ins., and should be so disposed as to allow of easy and efficient tamping. It should be well tied together vertically and laterally, if only with very light members.

Temperature Changes and Consequent Stresses.—No stresses are so certain to take place as are the stresses due to changes of temperature and to displacement of abutments. The experience of French and German engineers, who have built numerous masonry arches, has been such as to lead them to the use of hinges. While it may be a disputed question whether or not hinges should be used in steel-concrete arches, it is certain that temperature stresses must be taken care of.

Mr. Moisseiff.

It has been the writer's experience in calculating temperature stresses in concrete arches, and it is known to him to be the experience of other engineers who have calculated the same, that the thermal stresses reach high values, even for small ranges of temperature.

It is quite difficult to determine what range of variation of temperature should be allowed for. A mass of masonry will certainly not change its temperature very rapidly, and its range of temperature will not exceed the range between mean cold and mean warm weather. A range of 40° Fahr. would probably be sufficient to provide for. Even this low range will cause thermal stresses high enough to require careful consideration.

Mr. Worcester.

J. R. Worcester, M. Am. Soc. C. E.—One statement by Mr. Buck, if correctly understood, that there could be no comparison between the cost of steel-concrete bridges and steel bridges, seems to the speaker to be true only when one factor, that is, the cost of the abutments for the steel bridges, is left out of consideration.

The speaker's experience is, that, in the case of a highway bridge of moderate span, a comparison between a steel-concrete bridge and one of steel with buckle-plate floor filled on top, will show that the steel-concrete bridge is considerably cheaper; and that where good foundations are easily obtained, if the total cost of bridge and abutments is included in the comparison, there is a fair chance for steel-concrete construction to compete with all-steel.

Mr. Jacoby.

Henry S. Jacoby, Assoc. Am. Soc. C. E.—It seems now to be generally conceded that the theory of elasticity should be applied in designing steel-concrete and other masonry arches. While it has been customary to assume the moment of inertia of the cross-section to vary as the secant of the angle of inclination of the axis of parabolic arches, in order to simplify the determination of the horizontal thrust, no investigation of the relative effect on the design caused by this approximation has been published. It may, therefore, be of interest to call attention to the fact that some recent investigations show that the differences between the areas of cross-section obtained when the moment of inertia is assumed to vary in accordance with the law previously mentioned and those obtained when its true variation is considered are so small that they may be safely neglected in any design. As the difference in the labor involved in making the designs is so great, it is gratifying that the simpler method gives results which answer every practical purpose.

In regard to the amount of steel required, the speaker favors the plan adopted by Mr. Thacher, to use enough steel to resist the bending moments, and to let the concrete take the axial thrusts and the shear.

Attention is also called to the desirability of making some systematic observations of the deflections of existing steel-concrete arches

Mr. Jacoby. by those who may be conveniently located with respect to the larger arches in this country, such observations to extend throughout at least one entire year, so that definite data regarding their behavior under changes of temperature and live loading may be available.

Mr. Wason. LEONARD C. WASON, Assoc. M. Am. Soc. C. E.—The speaker has devoted his whole time for nearly eight years to the Ransome system of steel-concrete construction, and, as in fire-proof floors, the maximum stress in steel and concrete with minimum weight is necessary, especial attention has been given to the stresses in tension and in compression, the ratio of the steel section to the concrete section, and the proportions of cement and aggregates used. What follows is the result of personal experience, and, with the exception of the tests at the Watertown Arsenal, none of the data has ever appeared in print before.

Last winter the speaker made fourteen concrete beams for a student to test as thesis work at the Massachusetts Institute of Technology. The object of the tests was to determine accurately the strength of the combination, the position of the neutral axis of the beams, the maximum deflection at the first crack, and to see if the concrete cracked before the steel had reached its elastic limit. All beams were 12 ft. long, 8 ins. wide, and 12 ins. high, and were made as follows:

Beam.	Construction.
1st	Plain concrete.
2d	With $\frac{1}{4}$-in. bar at the bottom.
3d	" $\frac{1}{2}$-in. " " "
4th	" $\frac{3}{4}$-in. " " "
5th	" $\frac{7}{8}$-in. " " "
6th	" 1 -in. " " "
7th	" $1\frac{1}{4}$-in. " " "
8th	" two 1 -in. bars at the bottom.
9th	" two $1\frac{1}{4}$-in. " " "
10th	" two $1\frac{1}{4}$-in. " " " and one $\frac{1}{2}$-in. bar at the top.
11th	" two $1\frac{1}{4}$-in. " " " " two $\frac{3}{4}$-in. bars " "
12th, 13th, 14th	These were like Beams 9, 10 and 11, respectively, except that, in addition, fourteen U's of $\frac{1}{4}$-in. steel were imbedded in the beams, passing under the bars, and the ends projecting above the top of each beam.

All bars were put in straight, with the center 2 ins. from the top or bottom face of the beam. Many interesting facts were learned in these tests, but only such as relate to the subject in hand will be mentioned.

The component parts of the specimens were tested. The concrete in all cases consisted of one part by measure of Portland cement, three

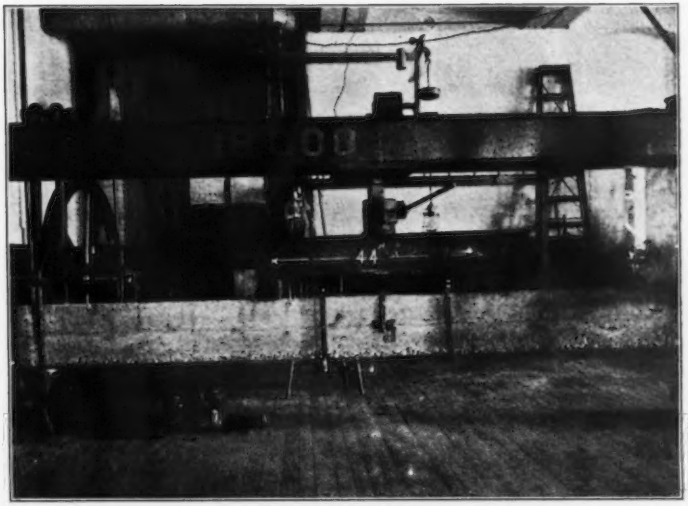


FIG. 1.—TESTING STEEL-CONCRETE BEAMS.

FIG. 2.—TESTING STEEL-CONCRETE BEAMS.

parts of Plum Island beach sand, and six parts of broken stone. The sand was perfectly clean; practically all passed through a 20-mesh sieve and was retained on a 30-mesh sieve; the grains, however, were round and smooth. The stone was a dark-blue trap; the pieces were cubical and the fractures angular. Four parts passed a 1¼-in. ring, two parts a ¾-in. ring. The steel was of a mild quality, square section, and was twisted cold until the edges made an angle of 20° with the axis of the bar. The cement was tested neat, and as 1 to 3 mortar. It developed nearly full strength in two weeks, so that there was but little increase in strength between 15 and 60 days. In the neat cement test, age 21 days, the average of five tension briquettes was 924 lbs.; highest, 978 lbs.; in compression, age 26 days, average of four, 8 140 lbs. per square inch; highest, 10 950 lbs. Mortar tension, age 12 days, average of four, 265 lbs.; the highest, 288 lbs. Mortar taken from concrete mixed by workmen, age 28 days, average of seventeen, 307 lbs.; highest, 368 lbs. Mortar by workmen, in compression, age 28 days, average of seven, 3 630 lbs.; highest, 4 500 lbs. Mr. Wason.

A concrete prism 8 × 8 × 12 ins. was also tested, and developed a strength of 2 455 lbs. per square inch in compression. A 3⅝-in. cube of stone failed at 22 000 lbs. per square inch. Some cubes of slightly larger size resisted the full power of the testing machine, and successfully withstood repeated stress of 20 000 lbs., both with and across the grain.

The average ultimate strength in tension on the steel bars of all sizes used was 83 330 lbs. per square inch, the highest being the ½-in. bars which carried 100 140 lbs. The average modulus of elasticity was 30 095 000. The elastic limit averaged 64½% of the ultimate strength.

The average weight of the concrete in the beams was 148.7 lbs. per cubic foot. The series was so made that the first specimen was quite weak in tension and the others were reinforced with steel until there was an excess of strength on the tension side, when the compressive portion was reinforced with steel to assist in its resistance to compression. The beams were all made in wooden forms inside the building, and left undisturbed until the time of these tests, which varied from 30 to 50 days.

The method used (Fig. 1, Plate VIII), was to rest the ends on small iron plates on the knife-edges of the testing machine, with a clear span of 11 ft. Blocks were laid on top of the specimen, dividing the span into three equal parts, and an I-beam was placed on top of these and the load applied through it. Thus a uniform bending moment was obtained between the two blocks. Yokes, 24 ins. apart, were attached rigidly to the middle portion of the beam, and micrometer readings were taken on the top and bottom as the beam was loaded. Assuming the plane of the yokes *A B* to be a plane, after the load was

Mr. Wason. applied, $A'\,B'$, we can find the position of the neutral axis as follows (see Fig. 1):

By similar triangles:

$$H : h :: x + y : y.$$

Therefore, $h = \dfrac{Hy}{x + y}$; H being the height of the beam, h the height of the neutral axis from the bottom of the beam, x the compression on the top, and y the elongation at the bottom. Clamps were attached to the beam over the supports, and fine piano wire was hung over these on either side of the beam, and, by means of micrometers attached to each side of the beam at the center, the deflection could be determined.

The plain concrete beam failed under a concentrated load of 2 500 lbs. at the center. Adding to this half the weight of the beam, namely, 600 lbs., and solving by the usual beam formula, the outside fiber stress is found to be 533 lbs. per square inch. The beam containing the ¼-in. bar failed at the same load. The bar pulled in two. If it is assumed that the bar was stressed to its limit, the concrete still had to carry 381 lbs. to the square inch in tension, as a concrete beam. The beam containing the ½-in. bar carried a greater load than was possible if the bar was required to carry the entire tensional stress. The bar pulled in two. If it is assumed that the bar was loaded to its ultimate strength, the concrete, as a beam, must still have supported part of the load, and the outside fiber stress would be 303 lbs. per square inch. Similarly, in the case of the ¾-in. bar, the concrete must carry 462 lbs. per square inch in tension. The ⅞-in. and 1-in. bars were capable of carrying the entire load coming on the beam containing them. The beam containing the 1¼-in. bar was accidentally cracked in two vertically in the middle in handling it before putting it into the machine, and carried 3 000 lbs. less concentrated at the center than the beam containing the 1-in. bar. As the 1-in. bar had an excess of strength on the tension side, there was no gain by the use of a 1¼-in. bar, and it is, therefore, safe to consider that the falling off in strength was due to the cracking of the concrete. On this assumption, the concrete in the beam with the 1-in. bar carried a stress of 516 lbs. per square inch. In the same way, the beam containing two 1¼-in. bars can be compared with the beam containing two 1-in. bars, which was accidentally cracked in two vertically in the middle; the ultimate strength of the former being 23 000 lbs. concentrated at the center, and of the latter 20 000 lbs. The 3 000 lbs. concentrated at the center would have to be carried by the concrete as a beam, and this would produce an outside fiber stress in tension of 516 lbs. per

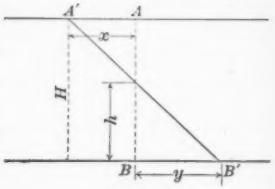


FIG. 1.

square inch, as in the previous case. The average of the foregoing results of maximum outside fiber stress in tension would be 466 lbs. per square inch. Mr. Wason.

The neutral axis was determined for each beam, and the total compression on the area above the neutral axis was calculated. This, in some cases, was easily done. With the $\frac{3}{4}$-in. bar, age 38 days, the maximum outside compressive stress was 2 916 lbs. per square inch; $\frac{7}{8}$-in. bar, age 50 days, 3 161 lbs.; 1-in. bar, age 50 days, 2 544 lbs.; $1\frac{1}{4}$-in. bar, age 41 days, 2 338 lbs.; and two $1\frac{1}{4}$-in. bars, age 42 days, 2 975 lbs.; the average of these being 2 787 lbs. per square inch. The beam containing two 1-in. bars (Fig. 2, Plate VIII), age 41 days, was cracked in two in the middle from top to bottom in handling, therefore the entire tension must have been carried by the steel and the entire compression was in the concrete. The neutral axis in this case was 10.2 ins. from the bottom, and this gave a maximum outside fiber stress of 8 588 lbs., which is very exceptional; yet, on account of being cracked in two, the stresses were more easily analyzed, and the result is correct.

In no case did a beam crack before the steel had reached its elastic limit. The average deflection in 11 ft. for twelve beams was 0.47 in., and the maximum was 0.76 in. when the first crack appeared. Every fracture was free from air holes, and showed dense, evenly-mixed concrete. In many cases the stone sheared in two without pulling from the mortar.

The formula used by the speaker for calculating concrete beams is:

$$f = \frac{W l}{6\frac{2}{3} D};$$

where W is the total load in pounds, l the span in inches, D the depth in inches from the top to the center of the bar, and f the total fiber stress in the steel. This formula was deduced at least twelve years ago. The proof follows:

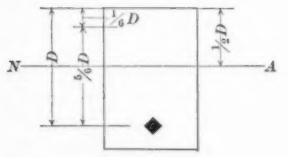


FIG. 2.

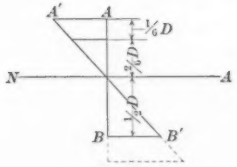


FIG. 3.

Assume as the fundamental condition an absolutely perfect bond of union between the steel and the concrete within the limits of the working strength of the combination. Assume that the steel takes the entire tensile stress, and the concrete the entire compressive stress. The concrete below the bar is for the purpose of holding it in place. While the concrete below the neutral axis certainly is in tension, it is

Mr. Wason. not considered in the computation. Assume the neutral axis half way between the center of the bar and the top of the beam. The center of pressure of the compressive area is two-thirds of the height from the neutral axis to the top, and the distance between this point and the center of the bar is five-sixths of the depth. This is the arm of the moment of internal forces, which is $\frac{5}{8}Df$. For a uniformly distributed load the moment of the external force would be $\frac{Wl}{8}$. Equating the two moments and solving for f, we get the formula,

$$f = \frac{Wl}{6\frac{2}{5}D}.$$

After solving and finding the total stress on the bar, a size and spacing can be selected which is convenient, and the area of concrete can be adjusted to resist the compression. The unit which the speaker has used, 16 000 lbs. per square inch in tension on steel, and for a mixture of concrete, 1:3:6, an average of 500 lbs. per square inch in compression on concrete, requires 32 sq. ins. of concrete in the upper third of the beam for each square inch of steel.

A good many pieces of actual work have been loaded with three times their calculated amount, and have shown no sign of failure whatsoever; therefore, structures calculated in this way are safe. The beams tested at the Massachusetts Institute of Technology carried from $4\frac{1}{2}$ to 5 times the load they were calculated to carry, according to the foregoing formula. In these tests, with all bars 2 ins. from the bottom, the neutral axis would be 7 ins. from the bottom. As a result of measurements on twelve beams, although there was considerable variation in the position of the neutral axis, the highest being 10.2 ins., the lowest 4.74 ins., the average was 7.06 ins., or sufficiently near to check the assumption made in the formula. This is an extremely simple formula for every-day use, and while it may not conform to profound theory, it certainly gives safe results in the design of steel-concrete structures. The speaker believes this formula can be used for any composite construction within the safe limit of the bond of union between the steel and the concrete.

Some experiments on the bond of twisted steel have been made, as follows: $\frac{1}{2}$-in. bars were imbedded in blocks of concrete 6 ins. square and which varied in length from 12 to 28 ins. The bars projected about 2 ft. from one end of the blocks. One face of the concrete bore against a plate, the bar passing through a hole in its center. A direct pull was applied to the bar, and in each case, when the length exceeded 21 ins., the bar broke outside the concrete without pulling therefrom. In these cases the ultimate stress on the steel averaged 81 835 lbs. per square inch. From the data thus obtained it is possible to determine the exact length for bars of other sizes, and when this limit is exceeded there is no danger whatever of the bars drawing,

and therefore the formula can be applied. If 16 000 lbs. is the working stress on the bar, it is only necessary to imbed it one-fifth of the distance as just determined in order to get the safe working bond required by the formula. Mr. Wason.

This formula can be applied to proportion the section of steel in a bridge. After determining the bending moment in the usual way, insert this in the formula, and use for the depth the distance from the center of the bar to the other face of the arch; then solve, and the size of the bar which will resist this moment is found. If other stresses, such as temperature, are reduced to the proper form, the steel can be proportioned in the same way to resist these stresses.

The neutral axis for each beam was computed from theoretical grounds, as follows:

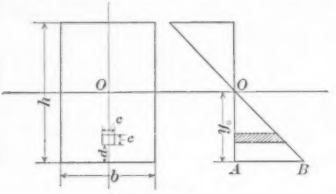


FIG. 4.

Let h = height of beam;
y_o = height of neutral axis from the bottom;
b = breadth of beam;
c = breadth of square rod;
d = height of rod from bottom of beam;
a = strain at one unit distance from origin;

$$ay_o = AB = \frac{f}{E};\ a = \frac{f}{Ey_o}.$$

At distance y from O, $a_1 = \frac{f}{Ey_o}y$. Stress in concrete $= Ea_1$; in steel $= E_1a_1$.

Beam with one rod in the bottom:

$$E\int_{-\frac{b}{2}}^{\frac{b}{2}}\int_{o}^{y_o} a_1\,dx\,dy - E\int_{-\frac{b}{2}}^{\frac{b}{2}}\int_{o}^{h-y_o} a_1\,dx\,dy$$

$$-E\int_{-\frac{c}{2}}^{\frac{c}{2}}\int_{y_o-c-d}^{y_o-d} a_1\,dx\,dy + E_1\int_{-\frac{c}{2}}^{\frac{c}{2}}\int_{y_o-c-d}^{y_o-d} a_1\,dx\,dy = 0.$$

$$Eb\int_{o}^{y_o} a_1\,dy - Eb\int_{o}^{h-y_o} a_1\,dy - Ec\int_{y_o-c-d}^{y_o-d} a_1\,dy + E_1c\int_{y_o-c-d}^{y_o-d} a_1\,dy = 0.$$

$$Eb\left[\frac{y_o^2}{2} - \frac{(h-y_o)^2}{2}\right] - Ec\left[\frac{(y_o-d)^2}{2} - \frac{(y_o-c-d)^2}{2}\right]$$
$$+ E_1c\left[\frac{(y_o-d)^2}{2} - \frac{(y_o-c-d)^2}{2}\right] = 0.$$

$$\frac{Eb}{2}(2hy_o - h^2) + \frac{(E_1-E)c}{2}[d^2 - (c+d)^2 + 2y_oc] = 0.$$

Mr. Wason.

Two rods in the bottom:

$$\frac{E\,b}{2}(2\,h\,y_o - h^2) + (E_1 - E)c\,[d^2 - (c+d)^2 + 2\,y_o\,c] = 0.$$

Assuming the modulus of elasticity for concrete to be 1 000 000 and for steel to be 28 000 000, the calculated neutral axis was lower in every case than the observed, the average height being 4.77 ins. from the bottom. Assuming the modulus to be 2 500 000, the average height of the neutral axis, as calculated by the foregoing formula, was 5.37 ins. from the bottom. These results differ too much from the observed to be reconciled with one another, and therefore the assumptions made in the deduction of the formula do not agree with practice.

Curves were plotted for each beam, using the deflections as ordinates and the bending moments as abscissas, and in every case a curve was obtained. The curve, however, as a rule, was quite flat. Nevertheless, the fact that it was a curve proves that Hooke's Law does not apply exactly to a composite beam of this type.

Five years ago the speaker had a contract, in the Fall River Public Library, in which it was necessary to bring a load of 150 000 lbs. upon a pier of concrete 15 ins. square, and not being able to find any printed data of tests on large specimens, he made a series of experiments* at the Watertown Arsenal on piers 1 ft. square and varying in length from 2 to 14 ft. There were three series. In the first he used all hand-mixed concrete, 1 : 3 : 6, of the proportion given previously for beams, and of practically the same quality. The second series was exactly similar, except that the concrete was machine-mixed; the third series was of leaner mixtures.

The specimens were all mixed in the open air in midwinter, and filled into vertical moulds, in an open building without heat, and were allowed to freeze and thaw, according to the weather, until tested, at an age of from 35 to 47 days. It is the opinion of the speaker, from observation and research, that had these specimens been made in midsummer and tested at the same age, they would have developed about twice the ultimate strength recorded herein. The longest specimens were tested first, and were put into the Emery machine in a horizontal position. The counterweight hung from the center did not balance the weight of the specimens themselves, so that there was a bending moment producing an outside fiber stress of 40 lbs. per square inch. Nevertheless, every specimen failed by direct compression, the longest ones showing the greatest strength, although they did not have as long a time to set as the short specimens. The average of the hand-mixed series was 989 lbs. per square inch; of the

* For full report of these tests, see the Report of the Watertown Arsenal for the year 1897, "Tests of Metals and Other Materials for Industrial Purposes," pp. 349-385.

machine-mixed, 1 098 lbs. per square inch, an advantage of 11% in favor of machine-mixing. Mr. Wason.

Readings of the compression were taken on all beams 6 ft. and more in length, some with repeated stress, some with a long lapse of time between stresses, and many interesting facts were developed. For instance, a load of 800 lbs. was put on the 12-ft. column, No. 8763, and the compression measured immediately. After a lapse of 5 minutes the beam had regained a small portion of its compression. The same was true of the 14-ft. column, No. 8759. With the 10-ft. column, No. 8766, exactly the reverse took place. The initial load in every case was 100 lbs. per square inch, and was increased by 100-lb. increments until the elastic limit was reached. The instruments were then removed and the specimens broken. After each application of load, it was removed, and the specimens regained their entire compression up to 500 lbs. per square inch in most cases, and above this point regained over two-thirds of the amount they were compressed, showing the material to be quite resilient.

Curves were plotted with the compressions as ordinates and the loads per square inch as abscissas, and these were quite flat, but still sufficiently curved to show that Hooke's Law does not apply closely, except within the limit of 700 lbs. per square inch. At this load the modulus of elasticity for hand-mixed concrete was 2 500 000 for an average of ten tests; on machine-mixed concrete it was 2 870 000 for an average of ten tests. The highest result of all obtained was 4 545 000. The third series, all machine-mixed, was made with specimens all 6 ft. in length, the first having a mixture of 1 cement, 3 of sand and 6 of stone, the crushing strength being 1 166 lbs. per square inch; second, 1:4:8 mixture, crushing strength 849 lbs. per square inch; third, 1:5:10, crushing strength 728 lbs. per square inch; fourth, 1:6:12, crushing strength 483 lbs. per square inch; fifth, 1:7:13, crushing strength 432 lbs. per square inch. A curve was plotted for these, using for ordinates a ratio of the volume of cement to the volume of concrete, and for abscissas pounds per square inch, and it was found that the points could be represented by a straight line, thus showing that, within these limits, the strength of the concrete is directly proportional to the amount of cement used.

Another test of the relative strength of hand and machine-mixed concrete might be mentioned. A contractor got into an argument with the supervising engineer on a piece of work over the quality of concrete mixed by a certain type of concrete mixer. This was settled by a test of 1-ft. cubes made at the Watertown Arsenal. The concrete was composed of 1 part of Giant cement, 3 parts of sand direct from the bank, and 5 parts of broken stone passing a 2½-in. ring, with all material passing a ½-in. ring screened out. The specimens were tested at 90 days. The machine-mixed specimens averaged 4 263 lbs. per

Mr. Wason. square inch; the hand-mixed, which were a little better in quality than is obtained in every-day practice, averaged 3 187 lbs. per square inch; an advantage of 25¼% in favor of machine-mixing. The machine in question was the Portable Gravity Concrete Mixer.

The speaker also made a groined vault which was as near as possible an eighth-scale model of a room 27 ft. 9 ins. by 33 ft. 6 ins., the specimen being 52 ins. long and 42 ins. wide. Arch ribs sprung diagonally from one corner to the other with a rise of one-tenth of the span. The arch vault sprung between them with a rise equal to that of the ribs at the center. The ribs were 1 in. wide and 2 ins. deep, the vaults ½ in. thick. The concrete was composed of 1 part cement, 2 parts sand and 4 parts stone grit. A load was applied at eight points near the crown, four on the ribs and four on the vault, and the specimen carried 26 605 lbs. before failing.

All the foregoing figures would indicate that a somewhat higher working stress can be used in concrete than has been customary. The speaker, in his practice, has been in the habit of using for a 1:3:6 concrete (his standard), 500 lbs. per square inch in compression, 80 lbs. per square inch in tension, and a ratio of steel area to concrete area of 1 to 32, using 16 000 lbs. as the stress in steel. While the foregoing experiments would indicate that still higher pressures are possible with safety, the speaker does not recommend any increase until these results have been checked by other experimenters. With the rapid increase in the quality and uniformity of Portland cement, he believes that in a short time it will be possible to use larger unit stresses than are now customary; or leaner mixtures, with the present unit stresses, without any danger whatever of exceeding safe limits of strength.

Mr. Johnson. A. L. JOHNSON, Assoc. M. Am. Soc. C. E.—While steel and concrete have been used in combination for hundreds of years, the real scientific genesis of steel-concrete construction is comparatively recent, and much experimental work yet remains to be done before the subject is fully understood. This work is being rapidly prosecuted, however, especially abroad, and it will probably not be long before the science is fully developed. We will then have our steel-concrete engineers as we now have our bridge engineers.

So far as the speaker has been able to discover, the first man to use these materials in a scientific manner, the metal being imbedded in the lower, or tension, side of the concrete, was W. E. Ward, who in 1875 constructed a building at Port Chester, N. Y.,* in which "not only all the external and internal walls, cornices and towers were constructed of béton" (the word concrete was not then in use), "but all the beams and roofs were exclusively made of béton, reinforced with light iron beams and rods."

In 1869 François Coignet, of Paris, took out letters patent on a

* *Transactions*, Am. Soc. Mech. Engrs., Vol. iv, p. 388.

Mr. Johnson.

combination of béton and iron rods, but he had no conception of the proper methods of using the materials. Neither had Monier, who, in 1876, built some flower-pots of béton and wire. He used the iron merely as a webbing, which was preferably placed in the neutral axis of the slab. For circular tanks or such constructions as placed the whole cross-section in tension this was not an improper method, but was, of course, of no value in resisting a bending stress.

Ward, however, had the real scientific idea, and built rods into the lower side of his concrete beams and floor joists much as is done to-day. He had no formulas to guide him, but relied upon his judgment.

In the following year, 1876, Thaddeus Hyatt, of England (recently deceased), began some experimental work on concrete and metal construction, his tests being made by Kirkaldy, and in 1877 he published a book giving the results of his experiments. The tests were interesting and of much value in the development of the science.

Perhaps the first approximately correct formulas for the strength of this class of construction were derived by Julius Mandl, in Germany, by the method of least work, and about the same time, in this country, by J. B. Johnson, M. Am. Soc. C. E., using another method, both giving the same equations.

Later, in 1898, L. A. Sanders, Engineer for the Monier construction at Amsterdam, published an elaborate treatise on the science of this construction.

All the foregoing investigators, however, took for the ultimate strength of the construction the proportionate elongation at which the concrete on the tension side would fail when no metal reinforcement was used. Given a certain modulus of elasticity for the concrete in tension, and a certain tensile strength, it seemed evident that this would fix the maximum proportionate distortion obtainable in the concrete. Then when the concrete had failed on the tension side it was assumed that the formulas would no longer be available, and that, even if the maximum strength had not been reached, the construction was in an unsafe condition owing to the exposure of the metal reinforcement.

The speaker believes he was the first to point out, at least in this country, the error of these assumptions, and that a section plane before bending was still a plane after the concrete on the tension side had passed far beyond its limit of strength. He had observed on tests made under his direction that the cracks which certainly ought to have been there were not visible even on a plastered surface with five to ten times the maximum proportionate distortion obtainable when no metal reinforcement was used.

It was his idea at this time that the metal served as an equalizer of slight variations in the strength at different places in the concrete,

Mr. Johnson. and that the cracks, though undoubtedly present, were so greatly distributed and divided up as to be too fine to be visible. This being the case, it seemed evident that for all practical purposes, at any rate, a section plane before the point was reached at which this rupture was supposed to occur would still be a plane after passing this point; in which case the same formulas would apply, it being only necessary to omit from the moment of resistance the moment due to the concrete in tension, which did not amount to very much anyway.

The next important contribution to the literature on this subject seems to have been made by M. Considère, Ingénieur en Chef des Ponts et Chaussées, Paris, who published* quite a long discussion on the subject, the basis of which was a series of tests made by him on specimens about 24 ins. long by 2½ ins. square, subject to uniform bending moment throughout their middle portion. The specimens were made of a Portland cement and sand mortar, some having metal imbedded and others of the same batch with no metal. The metal was round iron rods or wire, no transverse metal being used, to avoid complications in the stresses.

His studies embrace the following points: Graphical and algebraic determinations of the moment of resistance of armed pieces, influence of the quality of the concrete and of the armatures, the most economical percentage of metal, effect of bad workmanship, value of the factor of safety, and the utility of symmetrical armatures, by which is meant those running in two directions at right angles to each other such as is used in Monier constructions. M. Considère's article marks a distinct advance in the science. While it is impossible here to enter into a description of the experiments and a discussion of the paper—in fact, it contains much that is neither new nor of especial interest in this country—it is necessary to mention certain important results obtained, having as they do, an enormous influence on the scientific design of steel-concrete structures.

These results, as to which the evidence seems conclusive, are as follows:

1.—In armed concrete beams, the concrete on the tension side will submit, without rupture, to a proportionate distortion of from ten to twenty times that at which it would fail in an unarmed direct tension test. It will also have, during the additional period of distortion, a strength nearly equal to its maximum strength in direct tension.

2.—Severe interior stresses are introduced in armed concrete constructions where rich concrete mixtures are used, owing to the shrinkage of the latter during the first eighteen months if exposed to the air, or a corresponding swelling during the period if in submarine work. For neat cement this amounts to 2 mm. per meter, decreasing with the quantity of base material added.

* *Le Génie Civil*, February 4th to 25th, 1899.

Mr. Johnson.

3.—These interior stresses are, to a considerable extent, relieved by the slipping of the bars or rods in the concrete, as to which action he says his tests leave no room for doubt. The stress is also relieved in time, if cracks are not previously developed, by what he calls the tendency of the concrete to eventually yield somewhat to a soliciting force.

With regard to the determination of the first result, M. Considère subjected an armed beam to a bending giving 20 times the proportionate distortion in the concrete in tension at which it should have failed, and then subjected it to 139 repetitions of loadings where the proportionate distortion in the concrete ranged from $4\frac{1}{2}$ to 13 times the maximum obtainable from an unarmed piece. He then carefully cut out an annular ring of concrete surrounding one of the rods and found not only that it was not cracked, but that it was still capable of carrying nearly its maximum load.

It seems scarcely possible to obtain more conclusive evidence than this. It is certainly a great surprise to the speaker, who has always believed that the cracks were really there, though so fine as to be invisible.

The next result was determined from a neat-cement specimen, 1 x $2\frac{1}{2}$ ins. in cross-section, having a $\frac{4}{10}$-in. rod imbedded in the center. The rod was originally measured, imbedded in the concrete, and then after being immersed in water for some time, measured again. The cement was then stripped off and the length again determined, when it was found to be of the same length as originally, though having been materially longer on the second measurement. In this way it was ascertained that the rod had been subjected to a tensile stress of 17 070 lbs. per square inch, and the concrete to a compressive stress of 937 lbs. per square inch.

These values are very high, and render the advisability of the use of very rich mixtures in steel-concrete construction questionable.

The third result obtained, however, indicates that relief from this stress will probable be afforded on account of the slipping of the bars in the concrete, and the tendency of the concrete to gradually yield to a continuing force. The speaker fails to see any consolation whatever in this.

Accepting the reliability of these three results, and summing up their effect upon the design of steel-concrete structures, we arrive at the following conclusions:

1.—The maximum moment of resistance of steel-concrete beams should include the tensile strength of the concrete as one of the elements thereof, operating to almost its maximum capacity. There will be no cracks in the concrete on the tension side previous to the development of this maximum moment.

2.—On account of excessive shrinkage stresses, we should avoid, in

Mr. Johnson. steel-concrete construction, the use of very rich mixtures of concrete. The speaker would construe this as referring more particularly to the usual mortar mixtures rather than to the ordinary proportions in concrete work.

3.—To avoid the slipping of the metal in the concrete, dependence should not be placed solely upon the adhesion of the concrete to the surface of the metal; but the rods should be so constructed as to have an additional and more reliable form of bond.

The adhesion is very great originally, and entirely sufficient for our needs; but the slipping undoubtedly occurs through the weakening of this adhesion by vibrations and shocks to which, more or less, all structures are subjected.

It is entirely apart from the purpose of this discussion to present anything as to the characteristics or relative merits of the various styles of steel-concrete construction now on the market, many of which are protected to a greater or less degree. The foregoing conclusions, however, give reliable criteria for the determination of these merits.

That the concrete does not fail at all in tension, for proportionate distortions amounting to 20 times as much as it ought to be able to stand, is to the speaker the most interesting of M. Considère's results. As to the others, they only serve to substantiate previous impressions.

It is certain that the cracks do not show, but that they do not exist would be impossible of acceptance except on the strength of such elaborate and precise experiments as M. Considére has conducted.

To the speaker's knowledge, nobody as yet has confirmed the first result, and it may be that many engineers will wait for this before accepting it. For this reason the formulas following are based upon the concrete in tension being of no value in the moment of resistance of the cross-section. We are all the more willing to make this concession to the natural conservatism of careful engineers, since, after all, the effect is inconsiderable except where very rich mixtures are used, which, as has been shown, are not advisable in this class of construction.

Let E_s = Modulus of elasticity of steel, in pounds per square inch;

E_c = Modulus of elasticity of the concrete in compression, in pounds per square inch;

F = Elastic limit of steel, in pounds per square inch;

f_c = Compressive strength of concrete, in pounds per square inch;

b = Width of section considered, in inches;

a^2 = Area of bars, in square inches;

d = Spacing of bars, in inches;

$\frac{a^2}{d}$ = Amount of metal per inch in width, in square inches;

M_o = Moment of ultimate resistance of cross-section, in inch-pounds, for failure in tension; Mr. Johnson.

M_o^1 = Moment of ultimate resistance of cross-section, in inch-pounds, for failure in compression;

M = Bending moment of external forces, in inch-pounds;

W = Total load on beam, in pounds;

w = Uniformly distributed load, in pounds per square foot;

P_s = Total stress on metal in width b;

P_c = Total stress on concrete in width b;

λ_1 = Unit elongation of extreme fiber in compression;

λ_2 = Unit elongation of steel;

e = The distance from the bottom of the concrete to the middle plane of the metal, in inches.

As we cannot count on developing, at the maximum load, a stress in the metal much greater than its elastic limit, it is well to use steel with as high an elastic limit as is consistent with such ductility as may be necessary in the particular construction used. Though it is often possible in these constructions to break the metal reinforcement entirely in two, yet the high stress in the metal which this implies is not developed until long after the construction as a whole has attained its maximum carrying capacity.

It will be assumed, therefore, that the modulus of elasticity of the steel is constant up to the development of the value F, which is practically the elastic limit, at which time the construction as a whole has reached its maximum resistance.

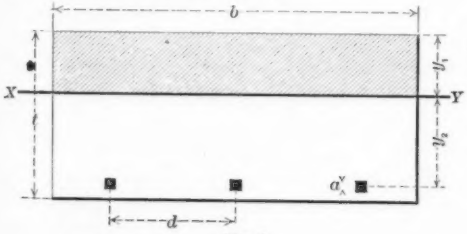


FIG. 5.

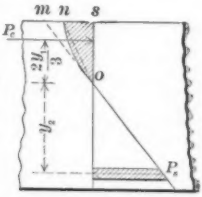


FIG. 6.

This condition does not hold on the concrete side. Here, the maximum load-carrying capacity will occur when the extreme fiber stress on the concrete becomes equal to its compressive strength, assuming that there is enough steel used to develop it. When this occurs, the stress diagram will be as shown above the neutral axis in Fig. 6. On this diagram the total stress P_c, equal to the area of the shaded portion, is about 25% greater than the area between the lines $o\,n$ and $o\,s$. The speaker has examined a large number of stress diagrams and finds that this is true within reasonably close limits for both rock and cinder con-

Mr. Johnson.

cretes of the usual mixtures. Also, in the same way, it was determined that the modulus of elasticity as represented by the tangent of the angle $n\,o\,s$ is in rock concrete only two-thirds and in cinder concrete one-half as much as the value of the original modulus, represented by the tangent of the angle $m\,o\,s$, which is the one usually given in test reports.

Single-Span Beams on Knife-Edge Supports.—Referring to Figs. 5 and 6 we have

$$P_s = \frac{F a^2 b}{d};$$

Then the strain, or stretch of the metal is,

$$\lambda_2 = \frac{F}{E_s},$$

$$\text{and } P_s = E_s \lambda_2 \frac{a^2 b}{d}.$$

On the compression side we have,

$$\lambda_1 = \lambda_2 \frac{y_1}{y_2}, \text{ and the resulting stress is,}$$

$$f_c = \lambda_1 \frac{2 E_c}{3} \text{ for rock concrete, and } \lambda_1 \frac{E_c}{2} \text{ for cinder concrete;}$$

$$= \lambda_2 \frac{y_1}{y_2} \frac{2 E_c}{3} \text{ for rock concrete, and } \lambda_2 \frac{y_1}{y_2} \frac{E_c}{2} \text{ for cinder concrete.}$$

$$\text{Also } P_c = \frac{5 f_c b y_1}{8} \text{ for both rock and cinder concretes.}$$

$$\text{Then } P_c = \frac{5 b y_1^2 \lambda_2 E_c}{12 y_2} \text{ for rock concrete,}$$

$$\text{and } = \frac{5 b y_1^2 \lambda_2 E_c}{16 y_2} \text{ for cinder concrete.}$$

But $P_c = P_s$; hence,

$$E_s \lambda_2 \frac{a^2 b}{d} = \frac{5 b y_1^2 \lambda_2 E_c}{12 y_2} \text{ for rock concrete,}$$

$$\text{from which } y_1^2 = \frac{12 a^2 y_2 E_s}{5 d E_c} \text{ for rock concrete.} \quad (1)$$

$$\text{Also } E_s \lambda_2 \frac{a^2 b}{d} = \frac{5 b y_1^2 \lambda_2 E_c}{16 y_2} \text{ for cinder concrete,}$$

$$\text{from which } y_1^2 = \frac{16 a^2 y_2 E_s}{5 d E_c} \text{ for cinder concrete} \quad (2)$$

For the moments of resistance of the cross-section we make no appreciable error if we assume that the location of the resultant pressure on the compression side, or the center of gravity of the upper shaded area in Fig. 6, is where it would be if the figure were a triangle.

Mr. Johnson.

We then have,

$$M_o = P_s\, y_2 + \frac{2\, P_c\, y_1}{3} = \frac{P_s}{3}\,(3\, y_2 + 2\, y_1) = \frac{F\, a^2\, b}{3\, d}\,(3\, y_2 + 2\, y_1) \ldots\ (3)$$

$$M_o^1 = P_s\, y_2 + \frac{2\, P_c\, y_1}{3} = \frac{P_c}{3}\,(3\, y_2 + 2\, y_1) = \frac{5\, b\, f_c\, y_1}{24}\,(3\, y_2 + 2\, y_1) \ldots\ (4)$$

We also have,

$$t = y_1 + y_2 + e \ldots\ (5)$$

The value e varies with the thickness and also the character of the metal. It should not be less than $\frac{1}{2}$ in. for bars, nor less than the thickness of the bar used. In such a material as expanded metal, in which the metal runs along planes inclined with the horizontal, this distance may be as small as $\frac{1}{4}$ in. In calculating any given construction the value e is first inserted in (5) which is then combined with either (1) or (2), depending upon which kind of concrete is used, to get the values of y_1 and y_2. These values inserted in (3) and (4) give the moments of resistance, the smaller of the two values being used as the strength of the section.

Single-Span Beams on Knife-Edge Supports of Equal Strength above and below the Neutral Axis.—Here M_o becomes equal to $M^1{}_o$ or,

$$\frac{F\, a^2\, b}{3\, d} = \frac{5\, b\, f_c\, y_1}{24}, \text{ from which } \frac{a^2}{d} = \frac{f_c\, y_1}{1.6\, F} \ldots\ (6)$$

Combining (1) and (6), and (2) and (6) we have,

$$y_2 = \frac{2\, F\, E_c\, y_1}{3\, f_c\, E_s} \text{ for rock concrete} \ldots\ (7)$$

and $$= \frac{F\, E_c\, y_1}{2\, f_c\, E_s} \text{ for cinder concrete} \ldots\ (8)$$

Introducing these values in (4), we have,

$$M_o^1 = M_o = \left[\frac{5\, b\, F\, E_c}{12\, E_s} + \frac{5\, b\, f_c}{12}\right] y_1^2 \text{ for rock concrete} \ldots\ (9)$$

and

$$M_o^1 = M_o = \left[\frac{5\, b\, F\, E_c}{16\, E_s} + \frac{5\, b\, f_c}{12}\right] y_1^2 \text{ for cinder concrete} \ldots\ (10)$$

In designing, where one is using a metal the quantity of which can be altered at will, then the construction should be made of equal strength in tension and compression, or $M_o = M_o^1$. In this case, if we desire to obtain a moment of resistance equal to a given bending moment, M, we have,

$M_o^1 = M$, and from (9) and (10)

$$y_1 = \pm \sqrt{\frac{M}{\frac{5\, b\, F\, E_c}{12\, E_s} + \frac{5\, b\, f_c}{12}}} \text{ for rock concrete} \ldots\ (11)$$

$$\text{and } y_1 = \pm \sqrt{\frac{M}{\frac{5\, b\, F\, E_c}{16\, E_s} + \frac{5\, b\, f_c}{12}}} \text{ for cinder concrete} \ldots\ (12)$$

Mr. Johnson. Equations (11) and (12) are very simple, as it will be noticed that, knowing the kind of concrete to be used, the radicals reduce at once to numerical fractions. When y_1 is thus determined we get $\frac{a^2}{d}$ from (6) and y_2 from (7) or (8). $\frac{a^2 b}{d}$ gives the amount of metal required for width b. The thickness of the concrete and the spacing of the bars will then only depend upon the size of the bars used.

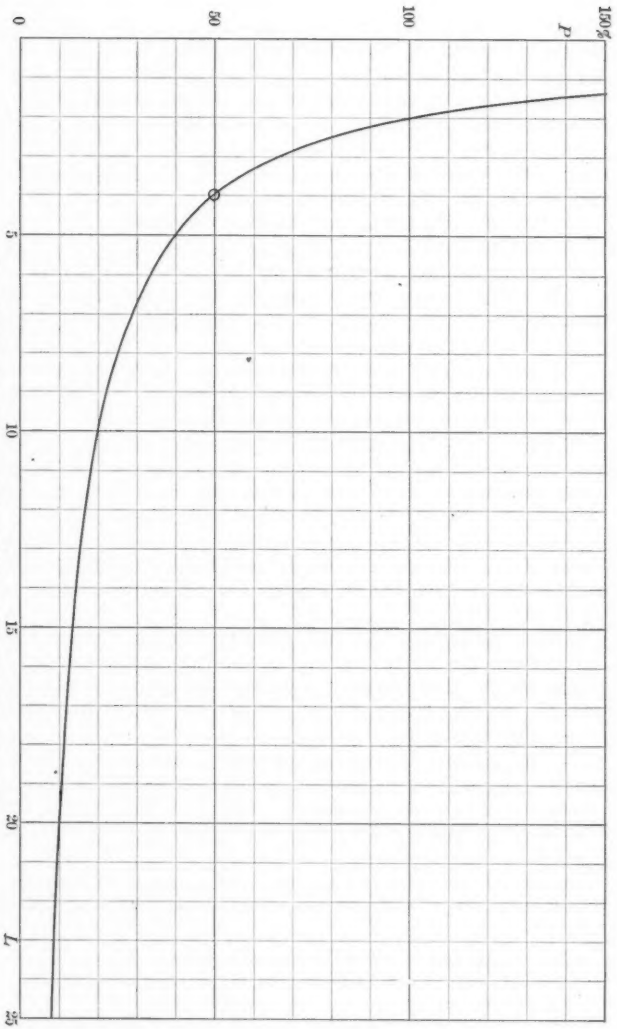
Floor Panels.—The foregoing formulas apply to steel-concrete, single-span beams, supported on knife-edges at the ends. The discussion is wholly inadequate when applied to the carrying capacity of such construction as is used in flooring where there are haunches on the beams and flooring in the adjacent panels. There has not, as yet, been developed a scientific discussion of the strength in such cases, and it is doubtful if there ever will be. It is known, in general, that a slab of a given thickness will carry more than twice as much, when incorporated in a floor, as it will when tested as a beam simply supported at the ends.

In fact, on spans of 4 ft. this ratio is more than 3 to 1, though probably this excess is entirely due to the fact that, in testing, a uniformly distributed load is generally used which will always arch considerably, no matter what the material may be. It is only fair, however, that the short spans should be allowed this advantage, as, when in use, the stored material cannot act otherwise. So far as concentrated loads in practice are concerned, except in special cases of rare occurrence, their effect is much over-estimated, and the special cases could receive individual treatment.

While, as has been said, it is, as yet, impossible to give a truly scientific discussion of these matters, we can arrive at an idea of the conditions by considering that the haunches on the beams produce a continuous-girder action, such that the external bending moment in the middle is only two-thirds as great as if it were free at the ends. Also that the floor in the adjacent panels produces an interior arching action within the thickness of the slab, adding about one-third to the area of the compressive stress diagram, this extra compression being counteracted by the adjacent floor panels instead of by the metal reinforcement. The effect of these two elements is to double the strength of the beams as used in floors.

On spans of 4 ft. in length uniformly loaded with any material not a grain or fluid, we can safely add another 50% to the moment of resistance on account of the arching effect previously alluded to. For zero span this effect would be infinite, and for infinite span it would be zero. We would then have, for the curve of correction to the moment of resistance, a hyperbola, as shown in Fig. 7, passing through a percentage of 50 at a span of 4 ft., and having the axes as asymptotes.

Mr. Johnson.



P = PERCENTAGE TO BE ADDED TO M_0 ON ACCOUNT OF THE ARCHING ACTION OF A UNIFORMLY DISTRIBUTED LOAD.
L = LENGTH OF SPAN IN FEET.

FIG. 7.

Mr. Johnson. Of course, the arching action varies with the deflection and also with the strength of the construction, the latter determining the height of the load. These are allowed for, in the speaker's experience with concrete and metal constructions, by the selection he has made of 50% at a span of 4 ft.

The whole argument as applied to floor panels is admittedly *a posteriori*, the speaker having had much experience with, at any rate, certain styles of construction, and knowing, therefore, in advance, about what his formulas should show. At least, the method has the merit of being safe.

The speaker's practice is, therefore, as follows:

For steel-concrete beams simply supported at the ends, use for the moment of resistance Formulas (3), (4), (9) and (10).

For floor construction, double the values given by the foregoing formulas and add to this a percentage depending upon the span length and obtained from Fig. 7.

For special cases of floor construction, where heavy concentrated loads are used, reduce the same to the equivalent non-arching uniformly-distributed load and then double the values in Formulas (3), (4), (9) and (10), and add no percentage.

The speaker has tables for the design of the special kinds of steel-concrete constructions used by him and for the concrete materials found in the territory of his operations.

As, however, these tables would not be applicable to other kinds of metal reinforcements, or to other localities where different kinds of materials are used in the concrete work, it is thought inadvisable to give them here.

It has been found that the elements of strength of concrete vary a great deal with different classes of materials, and the speaker would recommend that on any large work these functions be especially determined. This applies particularly to the compressive strength, and the compressive modulus of elasticity.

So far as floor construction is concerned, it is evident that the last word has by no means been said upon the subject of the theory of the combination of steel and concrete. But the speaker will be satisfied if he has shown that our knowledge is sufficient to give a safe and at the same time a fairly economical design.

As to simple beams, our knowledge is precise and sufficient.

So far as the other questions are concerned, with regard to the reliable use of these materials in combination, it would seem that they, too, have been practically all answered in the affirmative.

The coefficients of expansion are so nearly alike that no possible variations in temperature can produce any ill effects.

As to corrosion, bars may be imbedded in either rock or cinder concrete, if it is well mixed and has a reasonable quantity of cement, say

1:3:6, and left exposed to the elements for years and be as bright and clean when cut out as the day they were placed. Cinders containing a large percentage of sulphur will give just about as good a result, as these particles are covered over and rendered ineffective by the Portland cement water, or grout, that permeates the whole mass. Mr. Johnson.

As to its use in fire-proofing, where cinder concrete is used, the barriers have all been burned away, it having stood the severest test, both at home and abroad.

The admirable thing about this class of construction is the reliability with which a desired result can be obtained, and the performance again and again duplicated. One must, of course, know his materials. This is the quality that makes possible the development of the scientific theory, and renders it available for high-class engineering work.

The preceding part of this discussion applies particularly to flat-slab constructions rather than to deep beams. For such beams the value of the concrete in tension becomes of some consequence. It is impossible, however, to give a general expression for the moment of resistance, taking account of this element, without having an equation for the curve of the compressive stress diagram for the concrete used. Bach has derived such equations, exponential in form, and they have been used by L. A. Sanders, of Amsterdam, Engineer for the Monier Construction, in his pamphlet on "Cement-Iron Construction," published in 1899, of which the writer has made an English translation. But the equations were complicated and the treatment was incorrect in some particulars, especially with regard to fixing upon the proportionate distortion at which the concrete should fail in tension as the maximum capacity of the beam. This was known to be improper, even at that time, and later, M. Considère's results, previously referred to, gave precise information on this point.

Mr. Sanders handled his equations well, however, and demonstrated that it would be possible to take account of the curvature of the compressive stress diagram in the equations for the moment of resistance. To get even an approximately correct formula for the moment of resistance of a given construction made up haphazard, it would be necessary to introduce in the formula an equation expressing this relation between the stress and the strain, or f_c and E_c.

For the writer's part, he is not particularly interested in knowing how badly or how well other work has been designed, his chief concern being to try and design correctly himself; and for such cases the formulas can be simplified.

The correctly designed beam will be one of symmetrical strength; one equally likely to fail above or below the neutral axis. This means that the full compressive strength of the concrete will be developed, and that the full elastic limit of the steel will be reached.

When these two things occur it is known that the concrete in tension

Mr. Johnson.

has long passed the time at which its maximum strength was arrived at, which strength, nevertheless, is still intact.*

We then have, with reference to the concrete in compression, the same relations as those assumed in the first part of the writer's discussion. That is to say, we are using the whole of the compressive stress diagram, and while these diagrams vary much as to the angle the tangent makes at the origin, or the initial modulus of elasticity, yet the general relation of the total area of this diagram to the inscribed triangle, and of the original modulus to the resultant modulus for the maximum stress and strain, will not be greatly altered. By taking constants for these relations, which of course only apply to the maximum values of stress and strain, we obviate the necessity of introducing a complicating expression for these relations, which would, whoever, hold good for all points on the curve.

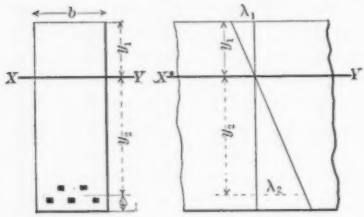


FIG. 8.

FIG. 9.

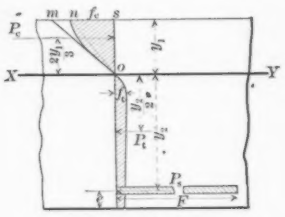


FIG. 10.

Fig. 8 is a cross-section of a steel-concrete beam of symmetrical strength.

Fig. 9 is the strain diagram showing the rectilinear variation of the strain across the section.

Fig. 10 is the stress diagram, showing the irregular variation in stress resulting from the uniform variation in strain across the section.

In addition to the symbols previously used,

Let f_t = tensile strength of the concrete, in pounds per square inch; P_t = total tensile strength of concrete for width b.

Then, as before,

$$P_c = \frac{5}{8} f_c \, b \, y_1 \ldots\ldots\ldots\

Mr. Johnson.

and $$\lambda_2 = \frac{F}{E_s}.$$

Then $$\lambda_1 = \frac{F\,y_1}{E_s\,y_2}$$

and $$f_c = \frac{2\,F\,E_c\,y_1}{3\,E_s\,y_2}, \text{ or } y_2 = \frac{2\,F\,E_c}{3 f_c\,E_s}\,y_1 \quad \ldots\ldots (14)$$

We also have for the steel:

$$P_s = \frac{F\,a^2\,b}{d} \quad \ldots\ldots (15)$$

For the concrete in tension:

$$P_t = \frac{8}{10} f_t\,b\,y_2 \quad \ldots\ldots (16)$$

The empirical constant, $\frac{8}{10}$, is obtained inferentially from the results of M. Considère. It cannot be much in error.

Then for the equation of the moments we have:

$$M_o = \frac{F\,a^2\,b}{d}\left(y_2 + \frac{2\,y_1}{3}\right) + \frac{8 f_t\,b\,y_2}{10}\left(y_2 + \frac{2\,y_1}{3}\right) \quad \ldots\ldots (17)$$

The equality of forces on each side of the section gives:

$$P_c = P_t + P_s \quad \ldots\ldots (18)$$

or $$\frac{5 f_c\,b\,y_1}{8} = \frac{8\,b f_t\,F\,E_c\,y_1}{15 f_c\,E_s} + \frac{F\,a^2\,b}{d} \quad \ldots\ldots (19)$$

from which $$y_1 = \frac{120\,F\,a^2}{75 f_c\,d - 64\,F\,d\left(\frac{f_t\,E_c}{f_c\,E_s}\right)} \quad \ldots\ldots (20)$$

or $$\frac{a^2}{d} = \frac{75 f_c\,y_1 - 64\,F\,y_1\left(\frac{f_t\,E_c}{f_c\,E_s}\right)}{120\,F} \quad \ldots\ldots (21)$$

also $$y_1 + y_2 + e = t \quad \ldots\ldots (22)$$

The stress on the concrete under the bar is not considered in these equations.

For a given depth of beam, Equations (14) and (22) will give at once the values of y_1 and y_2, and then the value of $\frac{a^2}{d}$ can be obtained from (21), which, inserted in (17), will give the moment of resistance for a symmetrical beam.

For the case of a given external bending moment we obtain from (14) the relation y_1 and y_2, this relation being only dependent upon the character of the steel and concrete used. Inserting this value of y_2 in terms of y_1 in (17) we obtain an equation in which the known external bending moment is equal to a function of y_1 and $\frac{a^2}{d}$. The relation between these is obtained from (21), which will at once give the value of y_1 and y_2, and $\frac{a^2\,b}{d}$ for the given moment.

Mr. Johnson. To illustrate the operation of the formulas, and at the same time to show how greatly the design is dependent upon the modulus of elasticity as well as the crushing strength of the concrete, the writer has taken a problem with two cases involving different values for these functions.

Problem.—Required, a steel-concrete girder beam of equal strength above and below the neutral axis, 15 ft. between supports, capable of carrying, as an ultimate breaking load, 2 000 lbs. per linear foot.

Here, neglecting the weight of the beam,

$$M = \frac{12\ Wl}{8} = 675\ 000 \text{ inch-pounds.}$$

For Case I, let $E_s = 30\ 000\ 000$ lbs. per square inch.

$E_c =$ 4 800 000 " " " "
$F =$ 50 000 " " " "
$f_c =$ 1 500 " " " "
$f_t =$ 200 " " " "
$b =$ 6 ins.

Then from (14) we have,

$$y_2 = \frac{2 \times 50\ 000 \times 4.8}{3 \times 1\ 500 \times 30}\ y_1 = 3.55\ y_1;$$

also, (17) becomes,

$$M_o = M = \frac{50\ 000 \times 6\, a^2}{d}\left(3.55\ y_1 + \frac{2\, y_1}{3}\right) + \frac{1\ 600 \times 6 \times 3.55\ y_1}{10}$$

$$\left(\frac{3.55\ y_1}{2} + \frac{2\, y_1}{3}\right) = \frac{1\ 266\ 000\ a^2\ y_1}{d} + 8\ 300\ y_1^2.$$

$$\text{But } \frac{a^2}{d} = \frac{75 \times 1\ 500\ y_1 - 64 \times 50\ 000\, y_1 \left(\frac{200 \times 4.8}{1\ 500 \times 30}\right)}{120 \times 50\ 000} = 0.00737\ y_1$$

Substituting this value of $\frac{a^2}{d}$ in the moment equation, we have,

$M_o = 17\ 625\ y_1^2 = 675\ 000,$
or $y_1 = 6.20$ ins.
Then $y_2 = 3.55\ y_1 = 22.0$ ins.,
and $t = 6.2 + 22.0 + 2 = 30.2$ ins.

$\frac{a^2 b}{d} = 0.00737\ b\ y_1 = 0.274$ sq. in., or slightly more than one, $\frac{1}{2}$-in. square bar.

We thus have the result that with the assumed values of the constants it will require a beam 6 ins. wide by 30 ins. deep, with one $\frac{1}{2}$-in. square bar imbedded, to carry 2 000 lbs. per running foot on a span of 15 ft.; which beam would be equally likely to fail by the crushing of the concrete above, or by the rupture of the concrete and metal below the neutral axis.*

* To this beam the exception made in the foot note on p. 122 would apply, and, in practice, more steel than this would have to be used. It serves the purpose of the present comparison, however.

Mr. Johnson.

For Case II, take $E_c = 2\,400\,000$ lbs. per square inch.
$f_c = \quad 2\,400$ " " " "
and all other values as before.

These values are by no means exceptional, it all depending upon the character of the materials used.

For this case,

$$y_2 = \frac{2 \times 50\,000 \times 2.4}{3 \times 2\,400 \times 30}\, y_1 = 1.11\, y_1,$$

instead of, as before, $3.55\, y_1$. This gives at once an idea of what may be expected.

$$M_o = M = \frac{50\,000 \times 6\, a^2}{d}\left(1.11\, y_1 + \frac{2\, y_1}{3}\right)$$
$$+ \frac{1\,600 \times 6 \times 1.11\, y_1}{10}\left(\frac{1.11\, y_1}{2} + \frac{2\, y_1}{3}\right)$$
$$= \frac{534\,000\, a^2 y_1}{d} + 1\,303\, y_1^2.$$

$$\frac{a^2}{d} = \frac{75 \times 2\,400\, y_1 - 64 \times 50\,000\, y_1 \left(\frac{200 \times 2.4}{2400 \times 30}\right)}{120 \times 50\,000} = 0.0264\, y_1$$

Substituting this value in the equation for the moment, we have:

$$M = 15\,403\, y_1^2 = 675\,000,$$

from which $y_1 = 6.62$ ins.
$y_2 = 7.35$ "
$e = 2.00$ "

and $t = 15.97$ " , or say 16 ins.

also $\frac{a^2 b}{d} = 6 \times 0.0264 \times 6.62 = 1.05$ sq. ins.

This, then, gives us a beam 6 ins. wide by 16 ins. deep, with one 1-in. square bar imbedded, as the equivalent of the beam in Case I, 6 ins. wide by 30 ins. deep, with one $\frac{1}{2}$-in. square bar.

Taking concrete at 25 cents per cubic foot in place, and metal at 3 cents per pound in place, and supposing the centering to be the same for each case, the cost of the first beam per linear foot would be 33.80 cents, and of the second, 26.86 cents. There are other conditions, also, which make the shallower beam preferable.

The foregoing will serve to show how important it is to know with what materials we are dealing, and it was for this reason that the writer in the first part of his discussion so strongly advocated determining the characteristics of the concrete for the particular material available on work of any consequence. To know the compressive strength, is not sufficient; it is just as important to know the modulus of elasticity in compression.

It is regretted that Mr. Wason did not give the center loads carried by the beams tested at the Massachusetts Institute of Technology, as

Mr. Johnson. the writer would have liked to apply his formulas to them; though, as the modulus of elasticity was not observed, it would not have been a very good test. From the fiber stresses which are given he has endeavored to argue back to this load, through Mr. Wason's premises, but it would seem that some error has been made in the calculation of these stresses, judging by the case of the ¼-in. rod, where both load and fiber stress were given.

Of course, the assumptions he has made in arriving at the fiber stresses are, as he himself says, erroneous, as are the formulas for the design of such beams. So, too, is the calculus discussion for the location of the neutral axis, it being based upon a uniform variation of stress across the section, which, as may be seen by reference to Fig. 3, is far from the truth.

Mr. Wason, in his work, in general, assumes the neutral axis to be half way between the center of the bars and the top of the beam. For the 12-in. beams tested at the Institute this gave the neutral axis for all beams, 7 ins. above the bottom. He says that by measurements taken on the beams, it was found that the position of the neutral axis varied from 4.74 to 10.2 ins. from the bottom, the mean being 7.06 ins., which he regards as a good check on the assumed location. One might as well say that the average number of inhabitants per square mile in the United States multiplied by the number of square miles in any given district, would give the population of that district. It might, but the chances would be very much against it.

The writer is disposed to regard the facts given by Mr. Wason as of much value, and hopes that he will supplement his discussion with further data with respect to the loads applied in the series mentioned, and other results of a similar nature.

The writer's specialty is not in the line of arch bridge construction; although he has worked up one or two designs for spans between 80 and 100 ft., which, however, were not built. These designs were for regular street service, the arch being from 10 to 12 ins. thick at the crown, with a rise of 1 in 8. Arches having a thickness of 3 ft. at the crown, with a span of 100 ft., such as recently witnessed by the writer, would be regarded by him as no application whatever of the steel-concrete principle of construction, the metal being practically thrown away.

Furthermore, the temperature stresses increase rapidly with the crown thickness, and are, naturally, severe where the latter is three times as much as it need be.

Mr. Parker. WILLIAM PARKER, M. Am. Soc. C. E.—The question of fire-proof construction has only been lightly touched upon in these discussions. It would seem that, as concrete is used so largely in fire-proof construction, more consideration should be given to the subject of the effect of fire on concrete, which, in the various fire-proofing systems,

is depended upon for strength, either when acting alone or in conjunction with metal. The metal which is depended upon for strength should be placed far enough from the surfaces exposed to heat, to insure the maintenance of a temperature which is not injurious to the strength of the metal and which will not cause undue expansion. Mr. Parker.

In the speaker's opinion, in some cases, this has not been done.

Just how far from the surface exposed to heat concrete will be seriously affected, and just what kind of concrete is best adapted for use on the surface, are the questions to be decided by the engineer or architect for the different conditions in his structure. So far as the speaker can learn, there have been but few carefully-made investigations to assist in deciding such questions, but it is his belief that concrete properly made, of either stone or cinders, and unprotected by plaster or otherwise, will not be destroyed by fire and water, such as it would be subjected to in a warehouse, more than 1½ ins. from the exposed surface, and this only when there is the concentration of a powerful fire-stream; and that after a depth of 2 ins. is reached there is no serious injury.

Cinder concrete is, no doubt, one of the best of non-conductors of heat, and, as it resists fire and water very successfully, it should be used whenever practicable.

These conclusions are from experiments on a small scale made by the speaker, and from observations of a few experiments on a large scale, as well as by reading what little he could find in print.

The thickness of concrete required for the protection of columns and long girders, particularly to prevent undue expansion, should be determined carefully by experiments, the temperature of both fire and beams or columns being recorded. The speaker can find scarcely a single record giving these data in a satisfactory manner.

The question of repairs should be borne in mind when designing. From experiments made by the speaker it seems practicable to replace a protecting layer of cinder concrete on a ceiling when it has become injured by fire and water. This question of repairs is one of special interest when considering the matter of insurance.

William B. Fuller, M. Am. Soc. C. E. (by letter).—The use of combinations of steel and concrete in various forms of engineering construction is meeting with increased favor as engineers become better acquainted with its possibilities; but this form of construction, or the use of any formulas relating thereto, should not be attempted without adequate experience. Mr. Fuller.

It is, perhaps, very fortunate that the development of this industry has fallen into the hands of companies controlling so-called patent rights for the use of the various combinations, as these companies have learned by costly experience that there is concrete of all grades, and that it requires a high order of skill to produce good concrete,

Mr. Fuller. such as will enable them to guarantee safe work. The results attained by such skilled mechanics are far ahead of average work, and very large allowances should be made if such work is to be done by others than skilled workmen under skilled supervision.

The secret of success with steel-concrete construction is skill in the selection and putting together of the materials to form a monolithic mass, and, until the methods of manufacturing concrete become more systematic and the results more uniform, each worker in this field must depend in a large measure on his own experience as to the quality and strength of the work his method will produce.

It is particularly unfortunate that all engineers of the older school have taught that concrete must be placed dry, or, at least, so as to just quake under heavy ramming, and that the stone used should be of a uniform screened size, as success in steel-concrete work requires a radical departure from such teachings, and the use of light rammers, the placing of mushy concrete, and the use of a uniformly-graded aggregate.